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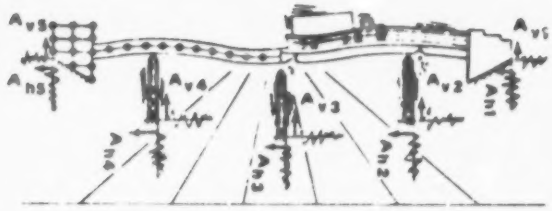
HIGHWAY STRUCTURE DAMAGE CAUSED BY THE TRINIDAD-OFFSHORE, CALIFORNIA EARTHQUAKE OF NOVEMBER 8, 1980

December 1981
Final Report

Earthquake Protection
of



Transportation Structures

Prepared for

U.S. Department of Transportation
Federal Highway Administration

Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

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FOREWORD

This report documents damage which occurred to highway bridges resulting from the November 8, 1980 Trinidad-Offshore, California Earthquake. Described herein are the estimated ground motion at the site of a collapsed bridge, details and the extent and cause of the bridge collapse, and the lack of damage to other nearby highway structures. The report was prepared for the Federal Highway Administration, Office of Research, by a member of the Earthquake Engineering Research Institute post-earthquake investigation team. It will be of interest to structural engineers and researchers interested in improving the earthquake resistance of highway structures.

Copies of the report are being distributed by memorandum to individual researchers. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

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16. Abstract

The moderate to severe (Richter magnitude 6.6 to 7.1) earthquake that occurred on November 8, 1980 approximately 40 km offshore of the Coast of Trinidad, California, having a focal depth of about 20 km, generally caused only superficial damage to a few structures but caused the collapse of a single freeway bridge. The report describes the estimated ground motion at the site of the collapsed bridge, details the extent and cause of the bridge collapse and describes the lack of damage to other nearby highway structures.

It was concluded that the collapse resulted from inadequate beam-bearing support of simply supported spans at both a pier and an abutment, and that the inadequate support was aggravated by the significant bridge skew. This bridge collapse is consistent with previous similar earthquake failures of bridges and further underscores the need to strengthen older bridges in seismically active regions.

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Introduction

A new task force for the specific purpose of conducting post-earthquake investigation of highway structures. The project, entitled "Seismic Damage Investigation and Evaluation," was conceived to improve the quality of earthquake engineering of highway structures through international observation of the performance of such structures subjected to moderate and severe earthquakes.

The post-earthquake investigation of the damage caused by the Trinidad-Offshore earthquake of November 8, 1980, was the first reconnaissance effort for this project. The damage to highway structures during this event was confined to a single site located approximately 350 kilometers (220 mi.) north of San Francisco. The Fields Landing Overhead, shown in Figure 1, suffered major damage with the collapse of two of its four spans. Two automobiles plummeted from the bridge, dropping 9 meters (30 ft.) to the roadway below, as shown in Figure 2. Six persons were injured, some seriously; five were members of a single family.

The structural connection detailing used in the design, coupled with the geometric requirements to accommodate railroad and local road alignment, resulted in a structure which was extremely vulnerable to seismic damage. Several other structures in the same area were also investigated and found to have suffered neither damage nor noticeable earthquake movements. Although these structures were basically of the same vintage and had similar characteristics, no single bridge contained every characteristic of the collapsed structure.

Pre-earthquake bridge maintenance records indicated that the Fields Landing Overhead was experiencing small movements due to temperature and shrinkage effects. The direction of movement corresponded to what was believed to be the failure mode caused by the earthquake. Also, previous earthquake damage to this bridge indicated an area of the structure which was particularly vulnerable to seismic motions.

The failure mode of this particular bridge reaffirms the lessons learned from past earthquakes; that is, structural details must be designed as part of the overall seismic resisting mechanism. Particular attention must be given to the forces and/or displacements that must be accommodated in order to mobilize a system's more ductile energy-absorbing components during a seismic event. Structures having details with deficiencies should be identified and prioritized for retrofitting through the use of vulnerability ratings. In addition, such vulnerability ratings should place the retrofitting needs in proper perspective with other highway safety funding needs so that managers less familiar with the seismic problems will have a basis on which to rationally allocate funding.

Examination of the methodology used to design the bridge and the apparent failure mechanisms substantiates the basis for the recent changes made by the California Department of Transportation in the seismic design approach to bridges. That is, elastic design spectra are used to evaluate overall response, with subsequent reductions to only ductile members. These changes are also reflected in the new provisions entitled "Guidelines for the Seismic Design of Highway Bridges," by the Applied Technology Council (ATC-6).

Seismological Aspects

The Trinidad-Offshore earthquake occurred on Saturday, November 8, 1980, at 2:27 a.m. (PDT) The University of California, Berkeley Seismographic Station reported a Richter magnitude of 7.0 for the event. Others reported Richter magnitudes ranging from 6.6 to 7.1. The epicenter of the earthquake was located, as shown in Figure 3, approximately 42 kilometers (26 mi.) west-northwest of Eureka, California, and had a focal depth of approximately 19 kilometers (12 mi.). Nine strong-motion accelerographs owned by the California Division of Mines and Geology (CDMG), Office of Strong-Motion Studies were triggered and recorded the earthquake [1]. Six of these stations recorded peak ground accelerations ranging from 0.05 to 0.15g. The duration of strongest shaking was about 10 seconds. A copy of the USGS computer output listing, giving locations and maximum recorded accelerations, is included in the Appendix.

The most intense ground motion occurred in the Fields Landing area, which is approximately 10 kilometers (6 mi.) south of Eureka. Fields Landing is approximately 46 kilometers (29 mi.) east-southeast of the epicenter. The damage reported within this area included the collapsed bridge, two homes that shifted off their foundations, and some minor damage to windows and light fixtures in the Eureka area. Minor damage extended to College of the Redwoods located approximately 12 kilometers (8 mi.) south of Eureka. "Reports from Kathy and David Jones, residents living within 100 meters of the bridge site, indicate a Modified Mercalli Intensity of VII."

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An Engdahl-Peak Shock Recorder installation at the Pacific Gas and Electric Humboldt Bay Power Plant indicates that the east-west peak ground acceleration there was about 0.26g. A plot of the three response spectra for 2 percent damping at this site is included in the Appendix. The power plant is located approximately 4 kilometers (2.5 mi.) north of the collapsed structure.

Another peak shock recorder, owned and maintained by the California Department of Transportation, Office of Structures, is located 6 kilometers (4.8 mi.) south of the collapsed bridge at the Loleta Overcrossing. Response spectra recorded for this site, which are also included in the Appendix, indicates a maximum ground acceleration of about 0.1 to 0.15g. From these observations an estimated ground acceleration of approximately 0.1 to 0.15g occurred at the bridge site during the earthquake.

Description of the Fields Landing Overhead

The Fields Landing Overhead, designed in 1959 and constructed in 1962, consists of two parallel reinforced concrete box girder bridges. A reinforced concrete slab resting on the two structures provides a median area, as shown on the General Plan in Figure 4. The four span bridges are simply supported, each measuring 123.8 meters (406 ft.) in length, with span lengths of 32, 32, 34, and 23 meters (105, 105, 112 and 75 ft.) The superstructure, which is on tangent alignment longitudinally, has large skew angles of 56 degrees at the abutments and bents.

The bridges share common "L" type abutments. Metal bearings, supported on concrete pedestals, as shown in Figure 5 are used at the abutments. The fixed bearing, shown in Figure 5a, is used at Abutment 1 (South Abutment) to provide a pinned connection. An expansion bearing, shown in Figure 5b, is used to provide a roller connection at Abutment 5.

Each bridge is supported on intermediate three-column bents. The columns are pinned at the superstructure soffit connection and fixed to a pile cap at the base. A 6-inch expansion joint hinge is also provided at each bent. A section view showing a cutaway portion of a typical bent is given in Figure 6. The hinge seats are interfaced with steel angles to provide a low friction, sliding surface for thermal and shrinkage movements.

The connections described above may be represented schematically for one of the bridges with boundary conditions, as shown in Figure 7.

The abutments, constructed on a 6.0 meter (20 ft.) deep roadway fill, are supported on 45-ton cast-in-place concrete piling. The column footings, constructed in original ground, are also founded on the same type of piling. The structure is located on an alluvial plane which extends 160 kilometers (100 ft) down to bedrock. The foundation material composition increases in sands from a relatively soft clay-like material near the surface to a dense compacted clayey material at a depth of approximately 120 kilometers (80 ft.)

Due to the soft near-surface foundation material, differential support settlement was a primary concern in the design of the bridge. The designer had to consider differential vertical movement at each of the supports, in addition to the normal horizontal temperature and shrinkage movements. Also, the large skew angle yielded a geometrically complex system with behavior which was difficult to model for a continuous superstructure with the then available analytical tools. These considerations resulted in the designers' final decision to design the bridge with the connection detailing described above as shown in Figure 7.

The Fields Landing Overhead was designed according to the American Association of State Highway Officials (AASHO) 1956, Seismic Design Criteria for 0.06g equivalent static loading for a structure founded on piling. Included in the equivalent static loading is an implied ductility factor of 4, which would yield an effective coefficient of about 0.24g maximum expected ground acceleration at the site for design of ductile components. The nonductile component may have been designed for an equivalent static force of 0.06g or simply included in the design as a simple connection detail.

Pre-earthquake Condition of the Bridge

Detailed historical maintenance records on the bridge [2] indicate that progressive relative displacements at the abutments and expansion joint hinges had occurred since the bridge was constructed. These relative displacements are primarily attributed to concrete shrinkage and thermal effects. Shown in Figure 8, superimposed on the schematic diagram used in the previous figure, are the relative displacements of the individual spans on the southbound bridge prior to the earthquake. In addition, some minor distress, as evidenced by concrete cracking, had occurred in the exterior girder stems of the superstructure in isolated regions near the expansion joint hinges, as shown in Figure 9. These regions are located at the acute corners in the bridge deck formed by the intersection of the skewed bent caps and

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the face of the exterior girder stems.

An earthquake occurred on June 5, 1975, which caused minor damage at the abutments [3]. Also some minor changes occurred in the accumulated relative displacements at the bearings and abutments, and additional cracking was documented in the distressed girder stems at the acute corners described above. Most of the 1975 damage occurred at Abutment 1. Reinforced concrete pedestals supporting the embedded steel bearings were also cracked. Records indicate that the degree of cracking ranged from minor to severe. Minor cracking occured on pedestals located near the east side of the abutment, with severe cracking evident at the westmost pedestals which support the southbound (i.e., collapsed) bridge. Keeper plates at three of the expansion bearings located at Abutment 5 suffered shear failure. The cracks in the abutment pedestals and girder stems were subsequently repaired by pressure injecting a concrete epoxy grout at a cost of $2,000.

Description of Damage to the Bridge

The Fields Landing Overhead, shown in Figure 10, suffered major damage during the earthquake. Two spans on the southbound bridge collapsed. In addition, all of the abutment bearings supporting the remaining portions of the two bridges were damaged.

All of the concrete pedestals at Abutment 1, which supported the collapsed Span 1 of the southbound bridge, suffered severe spalling on the top surface and vertical faces as shown in Figure 11. The embedded metal bearings were torn loose, shearing the two 1" diameter anchor bolts. The bearing pedestals at Abutment 1 of the northbound bridge were severely cracked, as shown in Figure 12. Also shown in this figure is the temporary timber shoring placed under the northbound bridge at Abutment 1 after the earthquake.

Abutment 5 experienced earthquake forces large enough to shear one keeper plate at each of the metal rocker bearings on both bridges. Figure 13a shows this type of failure at the leftmost bearing of the southbound bridge. Portions of the sheared anchor bolts, shown in the embedded base plate, were used to fasten the keeper plate. In each case, the sheared keeper plate was resisting movement to the left, which would have tended to rotate the superstructure, as viewed in plan, in a counter-clockwise direction. The total relative transverse movement of the southbound bridge, as indicated by scoring on the bearing plate, was approximately 4 centimeters (1.5 in.). It should also be noted that the total relative movement of the structure in the longitudinal direction was approximately 2.5 centimeters (1.0 in.), as indicated by the notch at the base of the rocker, shown in Figure 13b.

The hinge seat at Bent 3 was also damaged by the falling span, as shown in Figure 14. Note that the corner of the hinge bolster adjacent to the median was sheared, indicating a high concentrated load on this point just prior to the collapse. This would occur if the span, as viewed in plan, had rotated in the counter-clockwise direction.

Figure 15 is an aerial photo which shows the final position of the collapsed spans. Note that the hinged end of the spans had moved in an outward normal direction, indicating a combined translational and rotational mode of failure. The California Department of Transportation described the same basic failure mode in their report [4].

Investigation of Other Bridges

Eight other bridge sites located within an 8 kilometer (5 mi.) radius of the damaged Fields Landing Overhead were inspected. There was no damage to these bridges. Only minor indications of movement were discovered that could be attributed to the earthquake. The locations of the bridges inspected are shown on the map in Figure 16. Also, included in Table 1, is a tabulation of each bridge, giving its location relative to the collapsed bridge, its longitudinal direction, and the basic structural characteristics. Remarks relative to the field observations are also included in the table.

Spruce Point Overcrossing

This four-span continuous "T" beam bridge, measuring 69.5 meters (228 ft.) in length, crosses over US 101 approximately 5 kilometers (3.1 mi.) north of the damaged Fields Landing Overhead. A general plan of this bridge is shown in Figure 17. This bridge is supported on reinforced concrete piling with monolithically cast end diaphragm abutments and two column bents.

No evidence of permanent movement was found on the bridge; however, small gaps up to 6 millimeters (1/4 in.) were discovered between the soil and the column faces.

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Orchard Street Undercrossing

The Orchard Street Undercrossing is located 2.3 kilometers (1.4 mi.) north of the Fields Landing Overhead. As shown on the location map, this structure is the closest to the damaged bridge.

This undercrossing, shown in Figure 18, consists of two parallel three-span box girder bridges joined by a raised median slab. The bridges are 39.9 meters (131 ft.) long with hinges located in each of the end spans. The hinge details on these bridges are similar to those used on the damaged bridge. The bridges are supported on piling with separate end diaphragm type abutments and common five column continuous bents.

There is evidence that relative transverse and longitudinal movement between the two structures resulted from the earthquake. Historically, these two bridges experienced settlement problems, but increased gap openings at the expansion joints indicated that this bridge had also been affected by the earthquake. In addition, gaps measuring up to 6 millimeters (1/4 in.) were discovered between the soil and some of the column faces at Bent 2. There was also some minor cracking in the sidewalk under the bridge and in the soil surface between the sidewalk and Bent 2.

Hookton Road Overcrossing

The Hookton Road Overcrossing is located 4 kilometers (3 m.) south of the damaged bridge. It is the closest bridge on US 101 to the damaged structure. This two-span precast prestressed concrete girder bridge, as shown in Figure 19, carries traffic over US 101. The bridge is 50 meters (165 ft.) long and is on a curved alignment. The two-simply supported spans rest on a two-column bent and on end-diaphragm abutments, all of which are supported on piling.

Small gaps measuring approximately 5 millimeters (1/4 in.) were discovered in the Bent 2 deck expansion joints near the railing towards the outside of the curve. There was no other indication of movement at this bridge site.

Proposed Reconstruction and Retrofitting

The estimated cost to restore and upgrade the seismic resistance of the Fields Landing Overhead is $1,200,000. This cost includes: 1) demolition and removal of the two collapsed spans, 2) placement of earthquake restrainers in the existing undamaged portion of the superstructure, and 3) reconstruction of the collapsed spans, one three-column bent and abutment bearings. Because there was no damage to the railroad track, rail traffic resumed immediately following clean-up operations. Rail traffic was interrupted for approximately three days.

In an effort to remove the 6 kilometer (4 mi.) detour, an emergency contract was initiated on December 1st to upgrade the seismic resistance of the bridge that had not collapsed. This bridge will carry both the northbound and southbound traffic during reconstruction of the collapsed bridge. Restrainer devices designed to provide both longitudinal and transverse shearing restraint will be installed at each of the three hinged bents at a total cost of $50,000. As shown in Figure 20, steel cables will be used to provide the longitudinal restraint while steel pipes will provide the shearing restraint.

This design changes the existing structure because the hinge connection, capable of moving horizontally in any direction, now becomes pinned, capable of transmitting both longitudinal and transverse horizontal forces in the bridge deck. Temperature movements at the abutments are now accommodated for through the use of elastomeric bearing pads.

Concluding Remarks

The current CALTRANS criteria, based in part on the California Division of Mines and Geology map of "Maximum Expected Rock Acceleration from Earthquakes in California," places this site within the 0.5g contour, which requires an additional consideration of fault proximity. A maximum expected rock acceleration of 0.7g was used at this site for the redesign of the collapsed bridge. Using the newly prepared Applied Technology Council (ATC-6) publication, "Guidelines for Seismic Design of Highway Bridges," the acceleration coefficient is 0.3g, which yields for this site, including the effect of soil, a maximum ground acceleration of 0.45g [5].

The only significant damage to highway structures resulting from this earthquake was the collapse of the Fields Landing Overhead. The relatively low levels of acceleration and short duration of strong shaking reported for this earthquake indicates that this type of bridge, designed prior to modern seismic design practices, is particularly vulnerable to damage during a seismic event. Clearly in this case, the high skew coupled with the narrow bolstered expansion joint hinge led to the collapse of the two spans. Although girders at the acute corners of the expansion joint hinge were cracked prior to the earthquake, it is unlikely that this could have contributed to the collapse. Relative displacements that existed prior to the earthquake at the expansion joints, as well as previous undetected earthquake damage, may have contributed somewhat to the final

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collapse. Preliminary speculations, however, are that the failure was caused primarily by a combined response, which included both translation and rotation modes of vibration. In this case, the rotational mode was the dominant mode. Although the peak recorded ground accelerations were directed normal to the longitudinal axis of the bridge, it is doubtful that this had a significant effect, since the dominant mode is coupled in both horizontal directions.

Structures designed prior to the development of modern seismic design codes were designed for relatively low seismic coefficients, or force levels. In addition, implied ductility factors included in these coefficients are applicable only to the design of systems relying solely on ductile components for seismic resistance. The response of a bridge is not limited to forces experienced in ductile components, such as those evidenced in the Fields Landing Overhead. The total response of a structure composed of elastic, ductile and non-ductile components is best approximated by using elastic response spectrums. Unreduced seismic design coefficients or elastic design response spectra should yield slightly conservative, not grossly unconservative, design forces in the system employing elastic, ductile and non-ductile members in the seismic-resisting mechanism. The Fields Landing Overhead was designed using an equivalent static force coefficient of 0.06g, which yields a design force level lower than that experienced in this earthquake. In addition, the elastic displacements calculated at the hinge would be significantly smaller for the 0.06g design coefficient than it would be for an unreduced coefficient. The new codes for bridges have specifically addressed this problem by using an elastic design spectra for the response analysis. This anomaly was discovered following the San Fernando earthquake and incorporated in the 1975 California Department of Transportation Seismic Design Criteria, and more recently, in the ATC-6 Seismic Design Guidelines for Highway Bridges [5].

Failure of a single highway structure in a seismic event increases public awareness of unique structural engineering problems. In this particular case that awareness is focused on the narrow expansion joint hinges and the high skew. Since the San Fernando Earthquake of 1971, bridge engineers at the California Department of Transportation have been keenly aware of the vulnerability of this particular type of bridge. This is evidenced by the changes in the design of new structures and the retrofitting of existing structures. This bridge, which was slated for retrofitting, was awaiting budget approval by the local CALTRANS highway district. Having drawn public attention to this problem, it is important now that every effort be made to expedite retrofitting these vulnerable bridges. Fortunately, in this earthquake, vital lifelines were not disrupted and no fatalities occurred as a result of the bridge failure.

Recommendations

Highway structures, regardless of their lifeline status, should be rated for seismic vulnerability in a manner analogous to that used for traffic load ratings. This vulnerability index should eventually become part of the Bridge Inventory System and be used to prioritize seismic retrofitting and upgrading. Data gathered during routine or special maintenance inspections, which could be helpful in accessing or updating the seismic vulnerability, should be incorporated into this vulnerability index. In retrospect, there were several factors contributing to the failure of the Fields Landing Overhead which could have been input into a vulnerability index. This would have resulted in this structure being singled out and placed high on a priority list as a candidate for retrofitting. Both the biased movements due to temperature and shrinkage in the same direction as that of the failure mode and the earthquake damage caused in the June 7, 1975, Ferndale earthquake could have escalated this structure on a vulnerability list.

In addition to the basic consideration of seismicity and soil response, there are potentially two general categories to consider in establishing a vulnerability index. One includes the basic characteristic of the structure as designed and the other is the history of the structure's condition and response. Characteristics from each of the two categories include:

A. Structural Characteristics

1. Discontinuity at hinges

2. Bearing systems which become unstable after experiencing limited movements

3. The load capacity of non-ductile details (e.g., keeper plates, shear keys) are not sufficient to mobilize the ductile energy absorbing components

4. Skew effects on fundamental modes of vibration which are not reflected in the seismic analysis

5. Narrow expansion joint hinges

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B. Structural Condition and Response

1. Biased movements due to environmental effects and their correspondence to fundamental modes of vibration

2. Potential damage or deterioration resulting from past earthquakes

3. Earthquake damage from relatively minor events reflecting a response mechanism which was not anticipated in the original design

General purpose computer programs currently used for the seismic response analysis of bridges do not have the capability to model skew boundary conditions at intermediate hinges. Generally, the structural model is composed of space-frame members having six degrees of freedom at each joint. Skew boundary conditions are easily modeled at the abutments by specifying joint releases and their corresponding directions. However, the intermediate expansion joints, usually modeled as member end releases, are restricted to directions of the member coordinates oriented along the longitudinal axis of the bridge. Special purpose computer programs for the seismic analysis of bridges should either have these capabilities or special elements capable of modeling the properties for incorporation into general-purpose computer programs.

Bridge abutments that are generally constructed on fills could conceivably experience permanent horizontal and vertical displacements during an earthquake. Changes in span lengths or rotation at skewed abutments is a question which often arises in post-earthquake investigations. Established reference points on abutments or selected points on and off the structure would be helpful in detecting permanent displacements. Such reference on control points would be particularly helpful on bridges instrumented for seismic motions for post-earthquake correlation studies.

Unstable metal rod and rocker bearings should, as part of a retrofitting program, be replaced with more earthquake-resistant bearing systems. In situations where it is difficult or impractical to replace the devices, some means should be employed to prevent bearings from becoming unstable.

Highway maintenance engineers are generally concerned with expediting the normal flow of traffic to a damaged or impaired highway system. Post-earthquake investigators should be aware of this problem and make every effort to arrive at the damaged site as quickly as possible. If there is more than one damage site, the field investigations should be coordinated with the restoration operations, if possible, in order to conduct investigations prior to restoration operations.

The investigator should also try to determine the mode of failure as soon as possible, so that investigation may be directed toward gathering meaningful data prior to the demolition operation.

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TABLE 1
Compilation of Structural Characteristics of the Bridges Investigated Following the Trinidad-Offshore, California Earthquake of November 8, 1980

Location	Distance from Collapsed Bridge (kilometers)	Long. Direction	Type	Length (meters)	Number of Spans	Skew	Hinges	Abutments	Foundation	Field Observations
1 Spruce Point O.C.	5.0 north	N20°W	"T" beam	69.5	4	30°	none continuous	rigid diaphragm	concrete piles	Gaps in soil at columns.
2 King Salmon U.C.	3.7 north	N37°E	"T" beam	36.6	3	15°	none continuous	rigid diaphragm	concrete piles	No observed movements.
3 Orchard St. U.C.	2.3 north	N5°E	box girder	39.9	3	33°	2 adjacent to bents (similar to 4)	rigid diaphragm	concrete piles	Increased openings at expansion joints and gaps in soil beneath bridge.
4 Fields Landing O.H.	--	N3°W	box girder	124.0	4	56°	3 adjacent to bents	"L" type with steel rocker & rod bearings	concrete piles	2 spans collapsed and damage at abutment bearings
5 Hookton Road O.C.	4 south	N7°W	"I" beam prestressed precast	50.3	2	Varies 5° to 20°	1 at bent 2	rigid diaphragm	concrete piles	Small opening in rail - outside curve
6 Salmon Creek Br.	4.5 south	N12°E	"T" beam	47.6	4	10°	none continuous	rigid diaphragm	concrete piles	No observed movements.
7 Salmon Creek Br. (County)	4.5 south	N12°E	"T" beam	18.9	2	0	1 at center pier	simply supported spans cantilever wall	wooden piles	No observed movements.
8 Loleta O.C.	7.4 south	S89°E	"T" beam prestressed precast	49.2	2	18°	1 at center pier	"L" type with elasto-meric pads	concrete piles	No observed movements.
9 Singley Rd. U.C.	9.2 south	N80°W	"T" beam	44.0	3	20°	none continuous	"L" type with elasto-meric pads	concrete piles	No observed movements

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Figure 1
Aerial View of Collapsed Fields Landing Overhead (Photo-Neil F. Gilchrist, Times Standard - Eureka)

Figure 2
Photo Showing Automobiles After Plummeting Off Collapsed Bridge (Photo - Neil F. Gilchrist, Times Standard - Eureka)

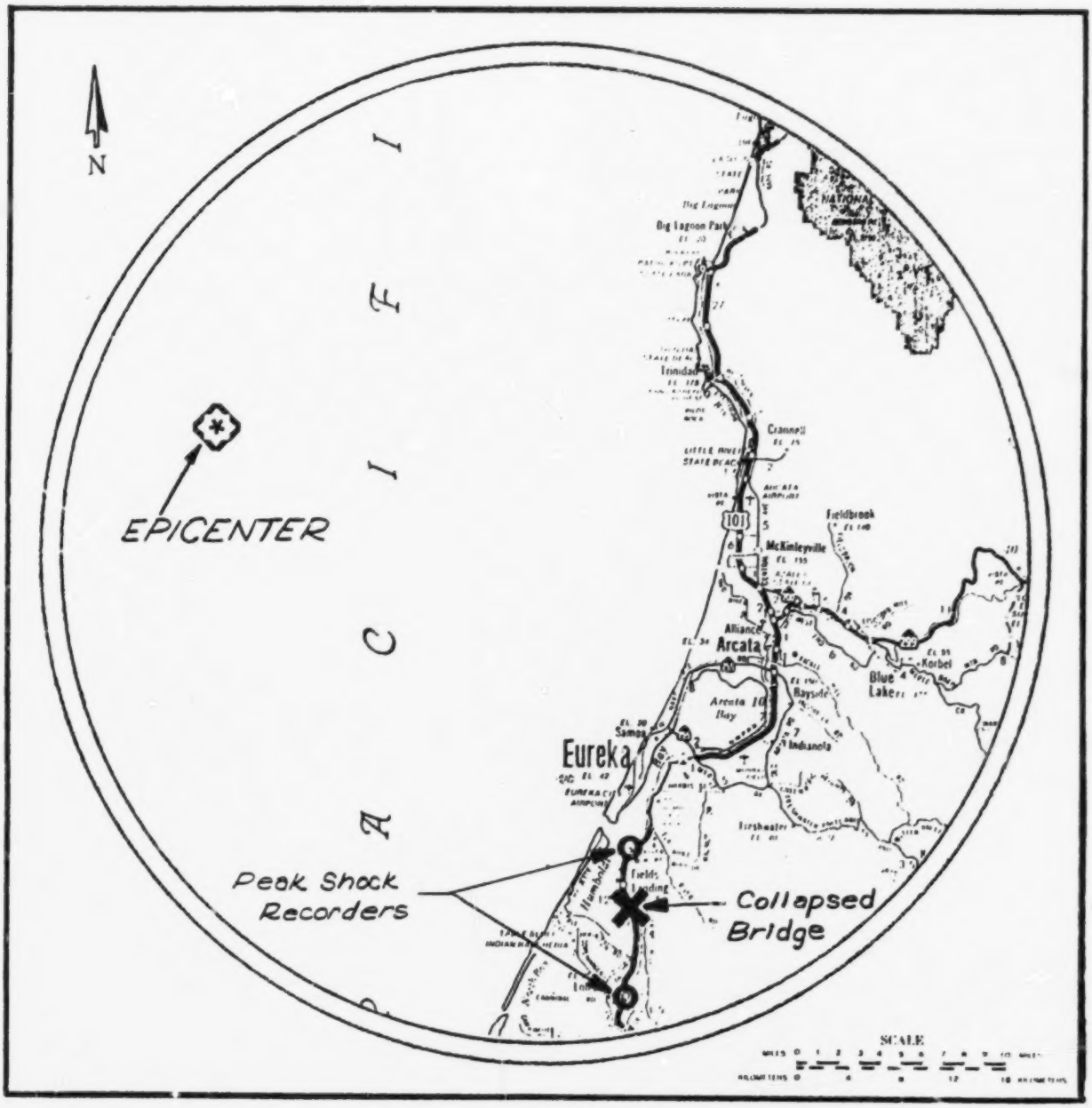

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Figure 3

Location Map

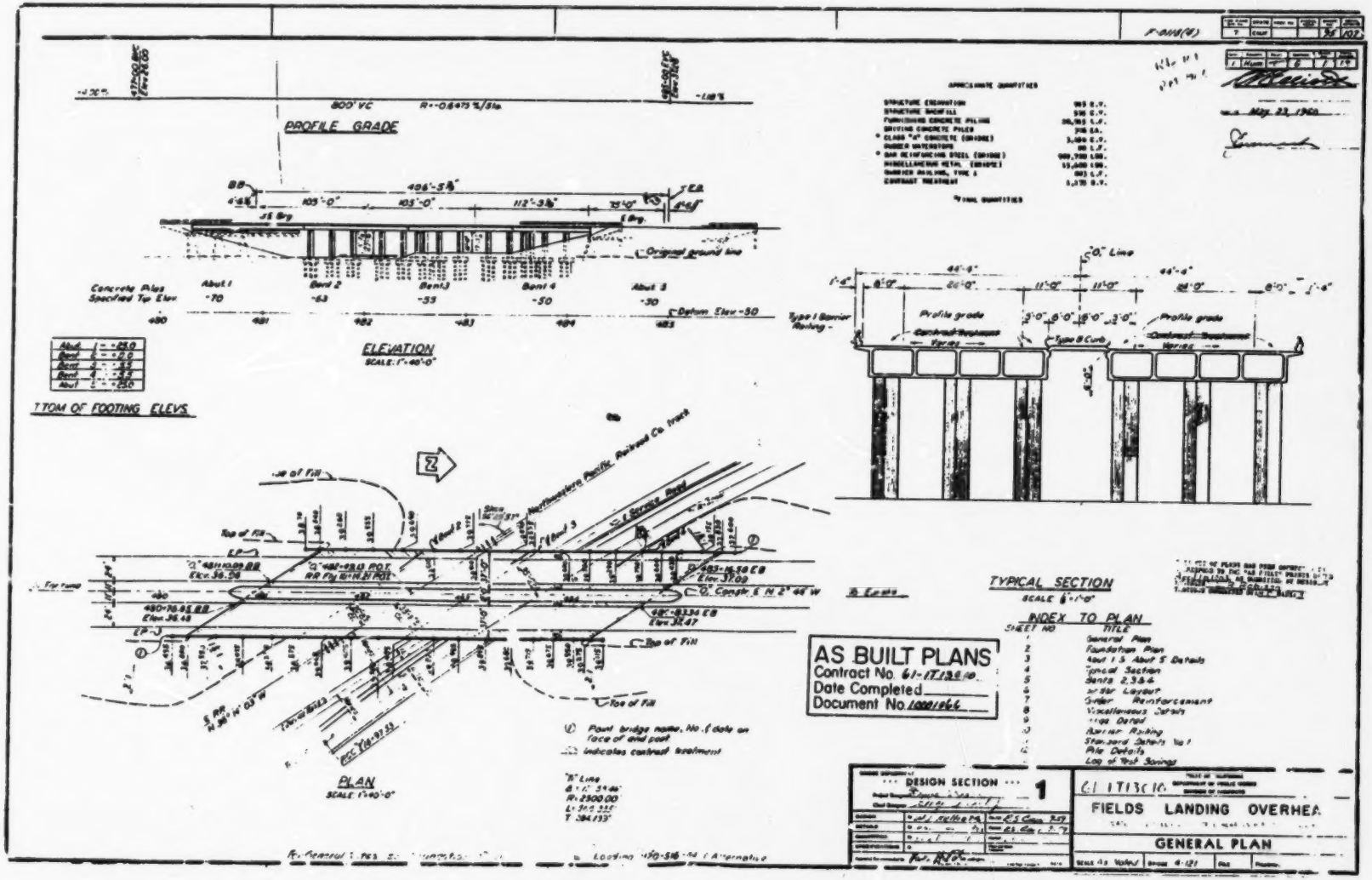

FIGURE 4
General Plan of the Fields Landing Overhead

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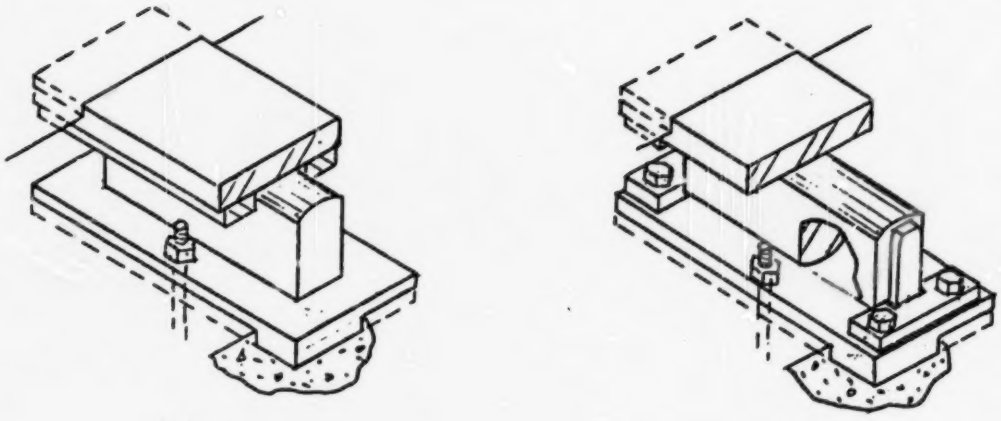


Figure 5

Abutment Bearing Details: (a) Fixed Bearing; (b) Expansion Bearing

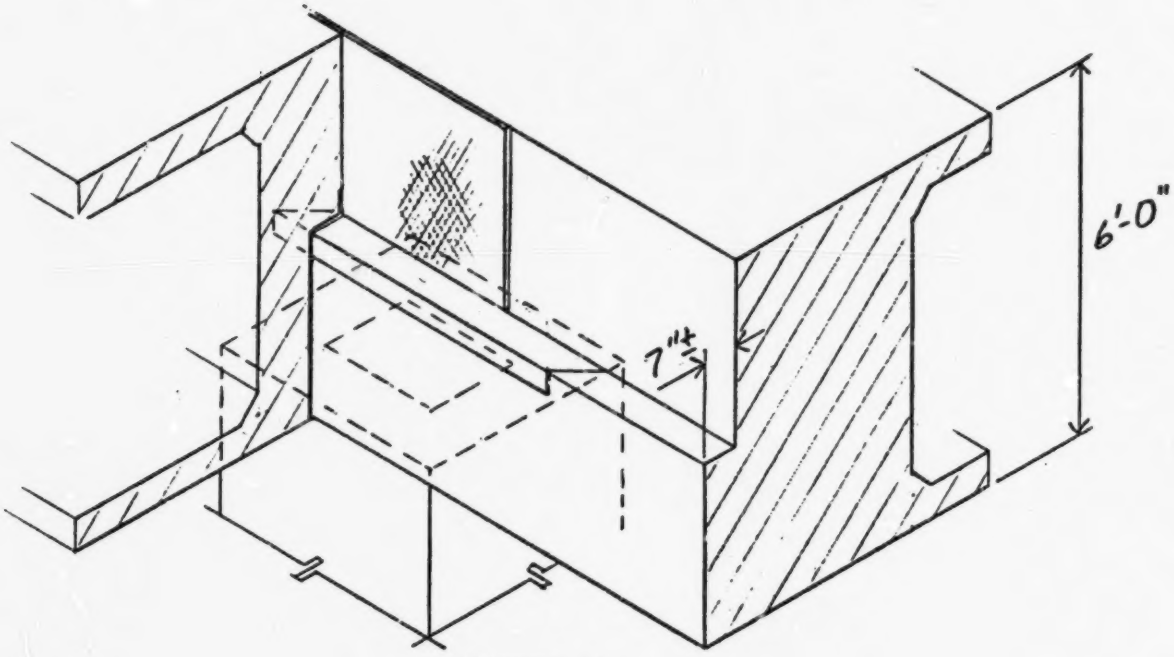


Figure 6

Section View at Bent Showing Expansion Joint and Pinned Connection at Column Top

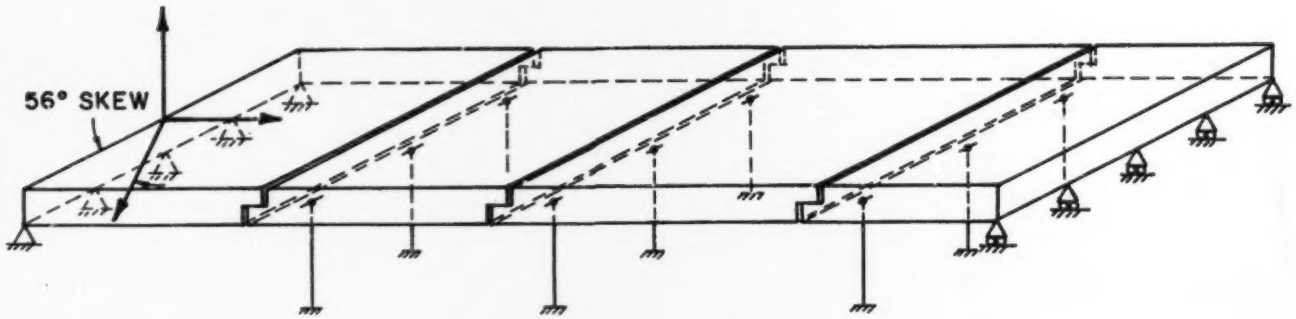


Figure 7

Schematic Illustration of Boundary Conditions
at the Abutments and Bents

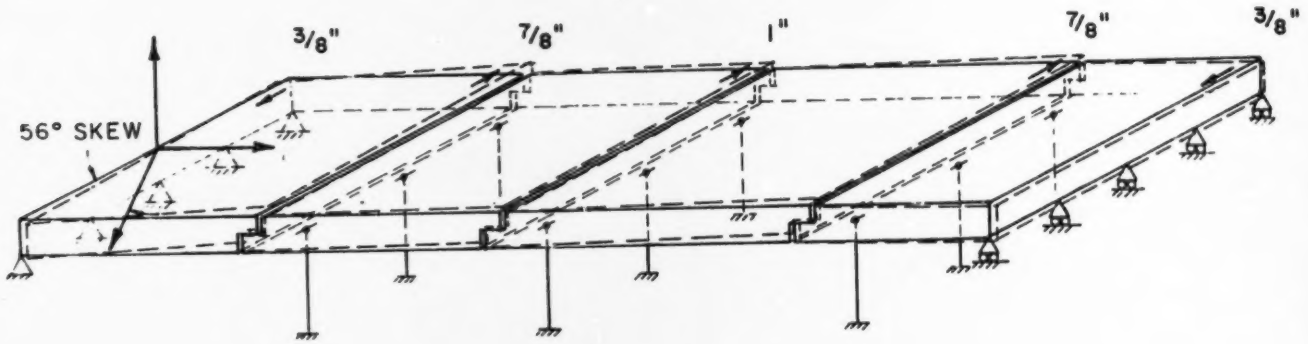
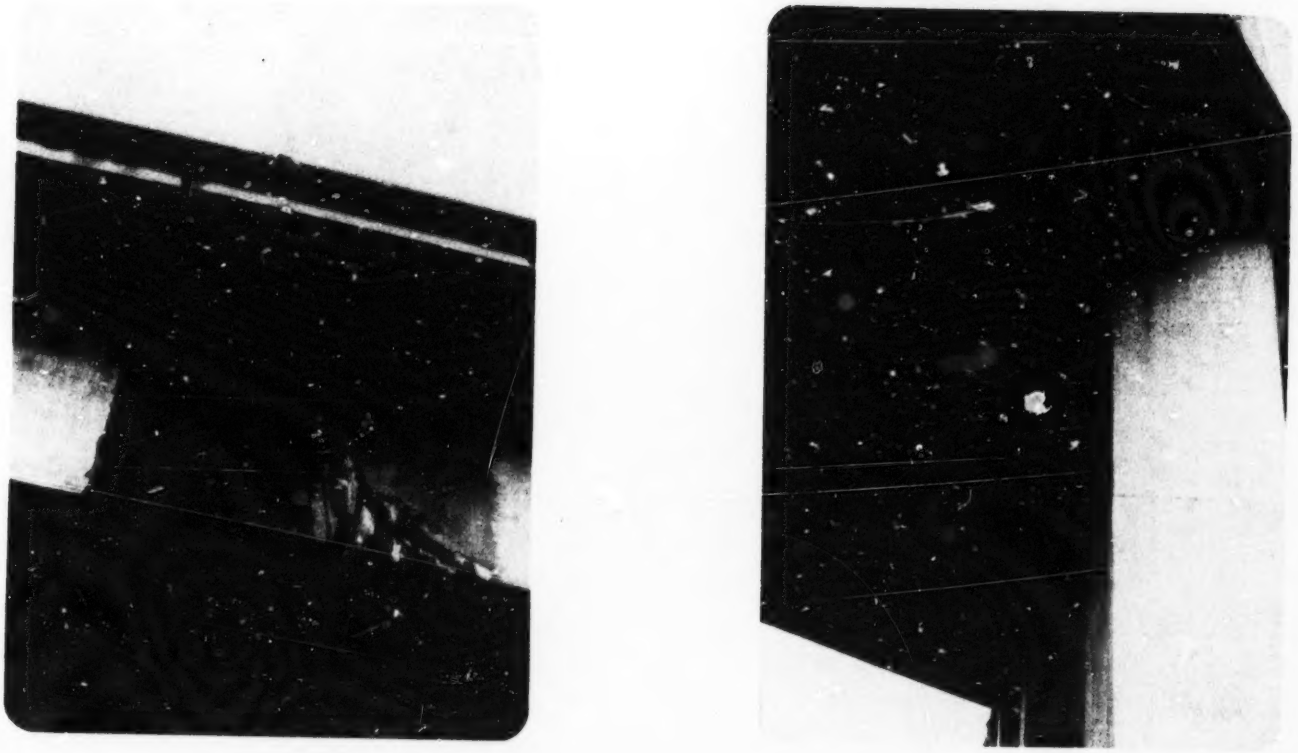


Figure 8

Relative Deck Offsets Prior to the Earthquake



a b

Figure 9

Distressed Pre-earthquake Condition of the Acute Corner of Bent 3 on the Northbound Bridge

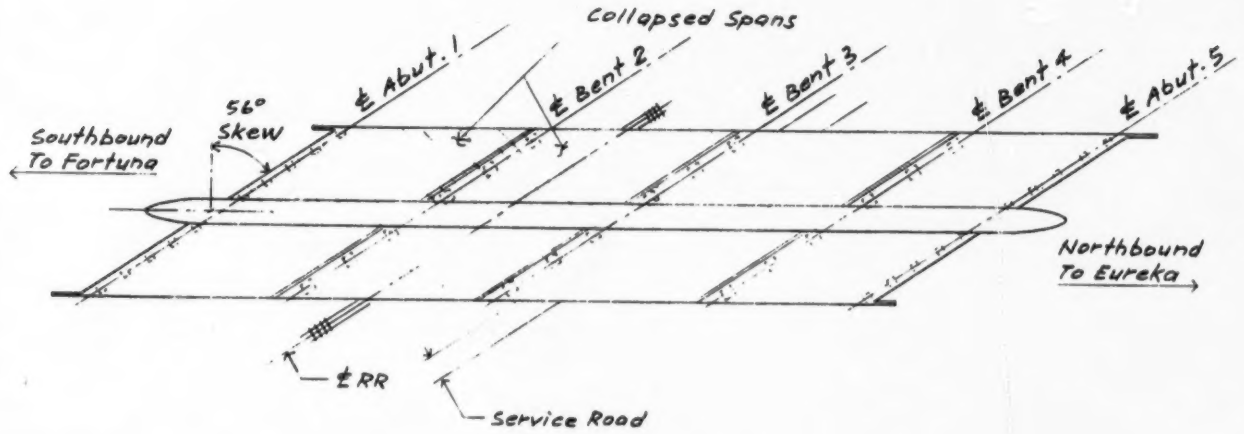


Figure 10

Plan View of the Fields Landing Overhead Showing Collapsed Spans 1 & 2 of the Southbound Bridge

Figure 11

Damaged Bearing of
Abutment 1 of the
Southbound Bridge

Figure 12a

Figure 12b

Damaged Concrete Bearing
Pedestals at Abutment 1
Northbound Bridge

Figure 12c
(Continued)

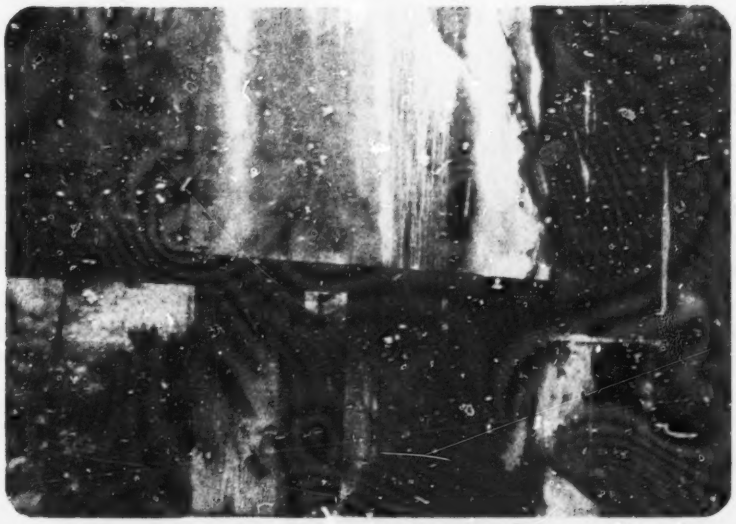


Figure 12d
(Continued)

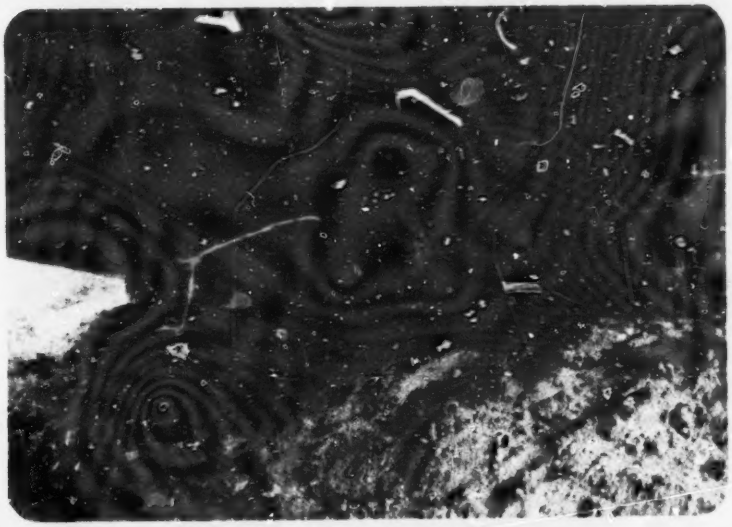


Figure 13a

Damaged Bearing at Abutment 5 Southbound Bridge: a) Sheared off keeper plate and scored surface indicating maximum transverse movement,

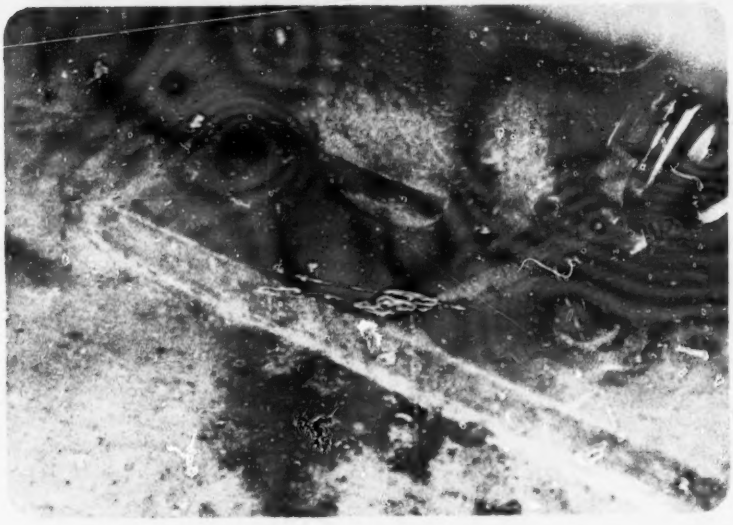


Figure 13b
(Continued)

Damaged Bearing at Abutment 5 Southbound Bridge: b) Notch in base of rocker indicating maximum longitudinal movement.

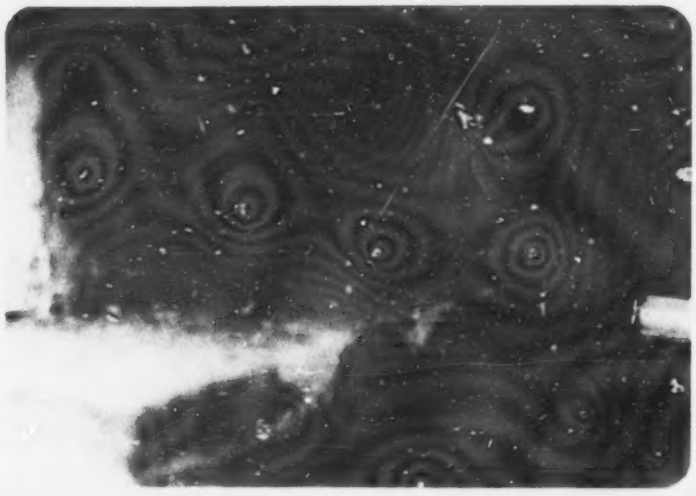


Figure 14

Damaged Hinge at Bent 3 of Southbound Bridge

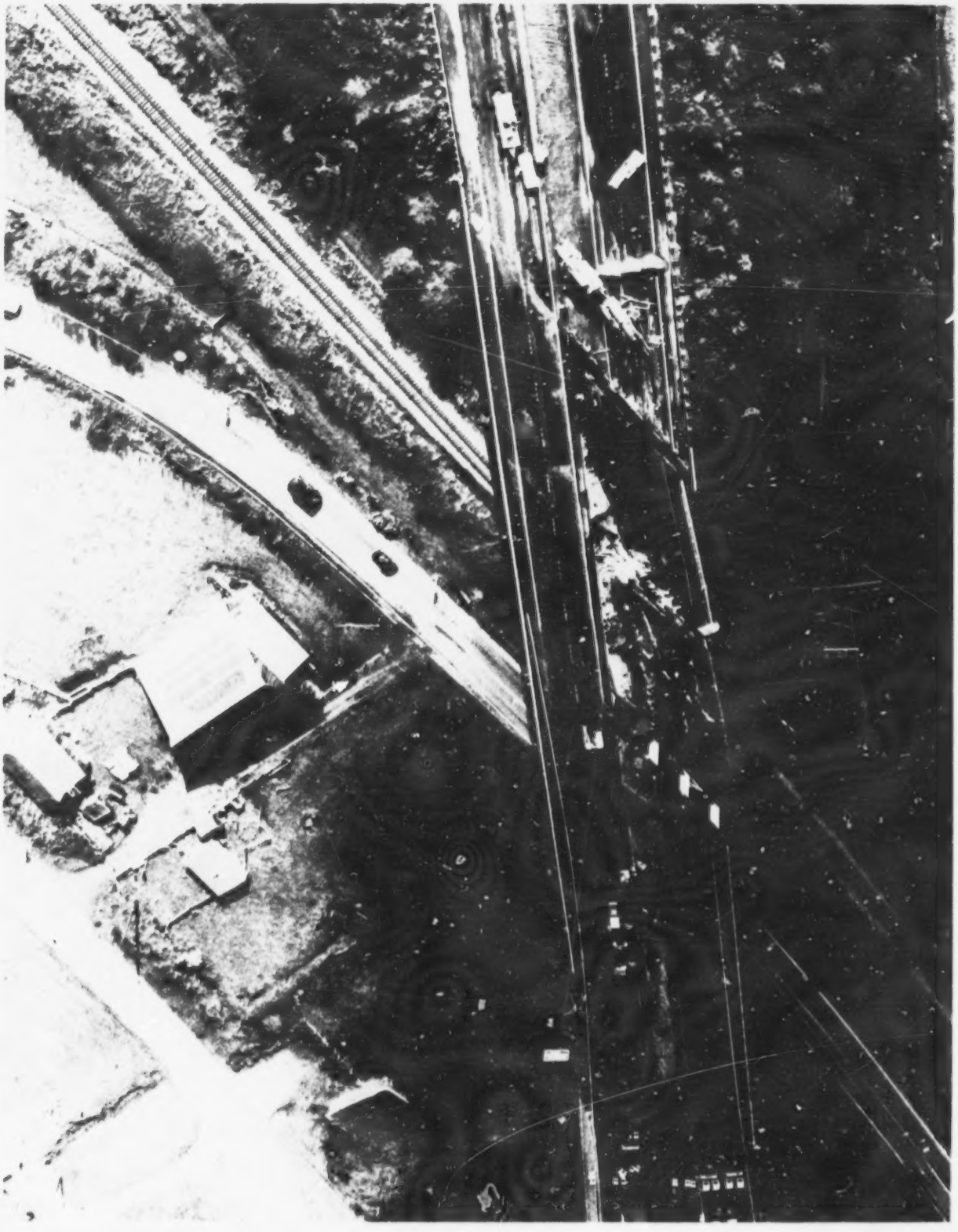
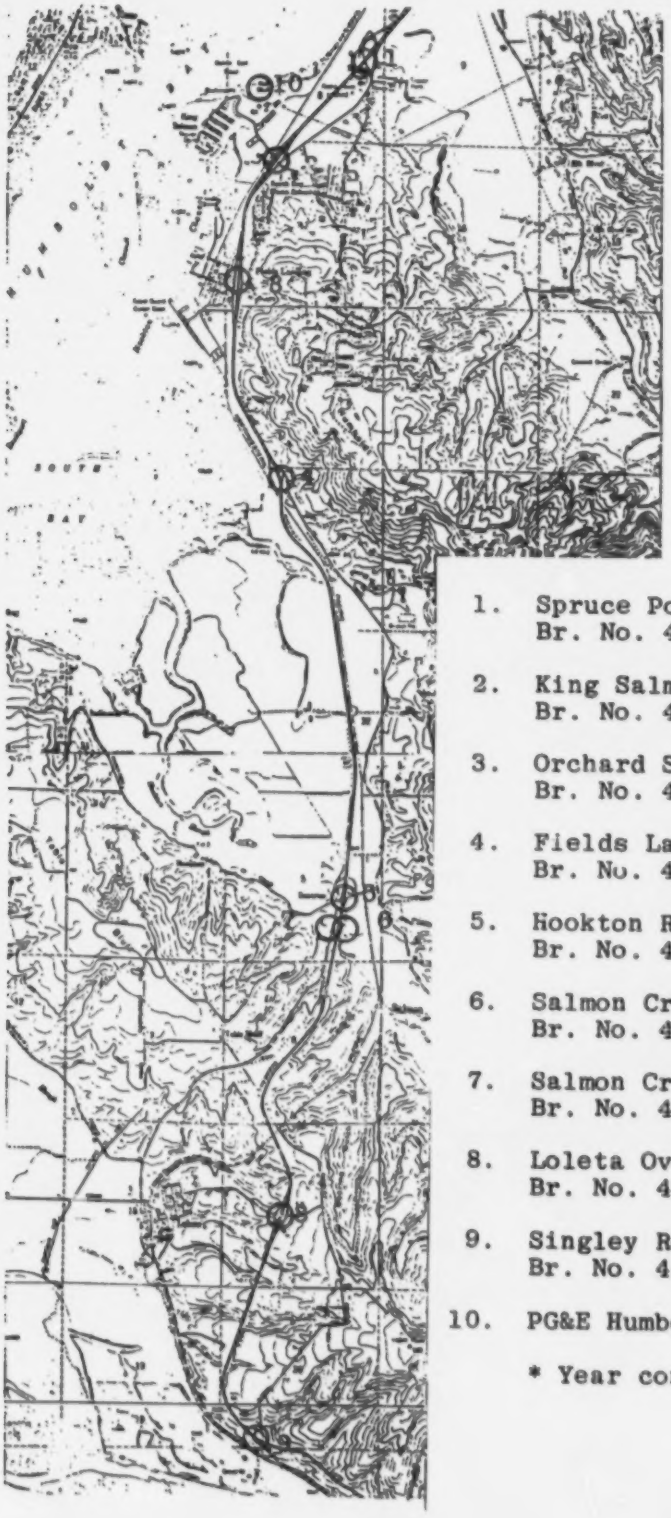


Figure 15
Aerial View of Collapsed Southbound
Lanes of Fields Landing Overhead

5.0 kilometers north to Eureka



1. Spruce Point Overcrossing
   Br. No. 4-71, 1963*
2. King Salmon Undercrossing
   Br. No. 4-67, 1963
3. Orchard Street Undercrossing
   Br. No. 4-122, 1963
4. Fields Landing Overhead
   Br. No. 4-121, 1963
5. Hookton Road Overcrossing
   Br. No. 4-166, 1962
6. Salmon Creek Bridge
   Br. No. 4-20, 1957
7. Salmon Creek County Bridge
   Br. No. 4-92, 1928
8. Loleta Overcrossing
   Br. No. 4-165, 1962
9. Singley Road Undercrossing
   Br. No. 4-110, 1958
10. PG&E Humboldt Bay Power Plant

* Year constructed

Figure 16
Location Map

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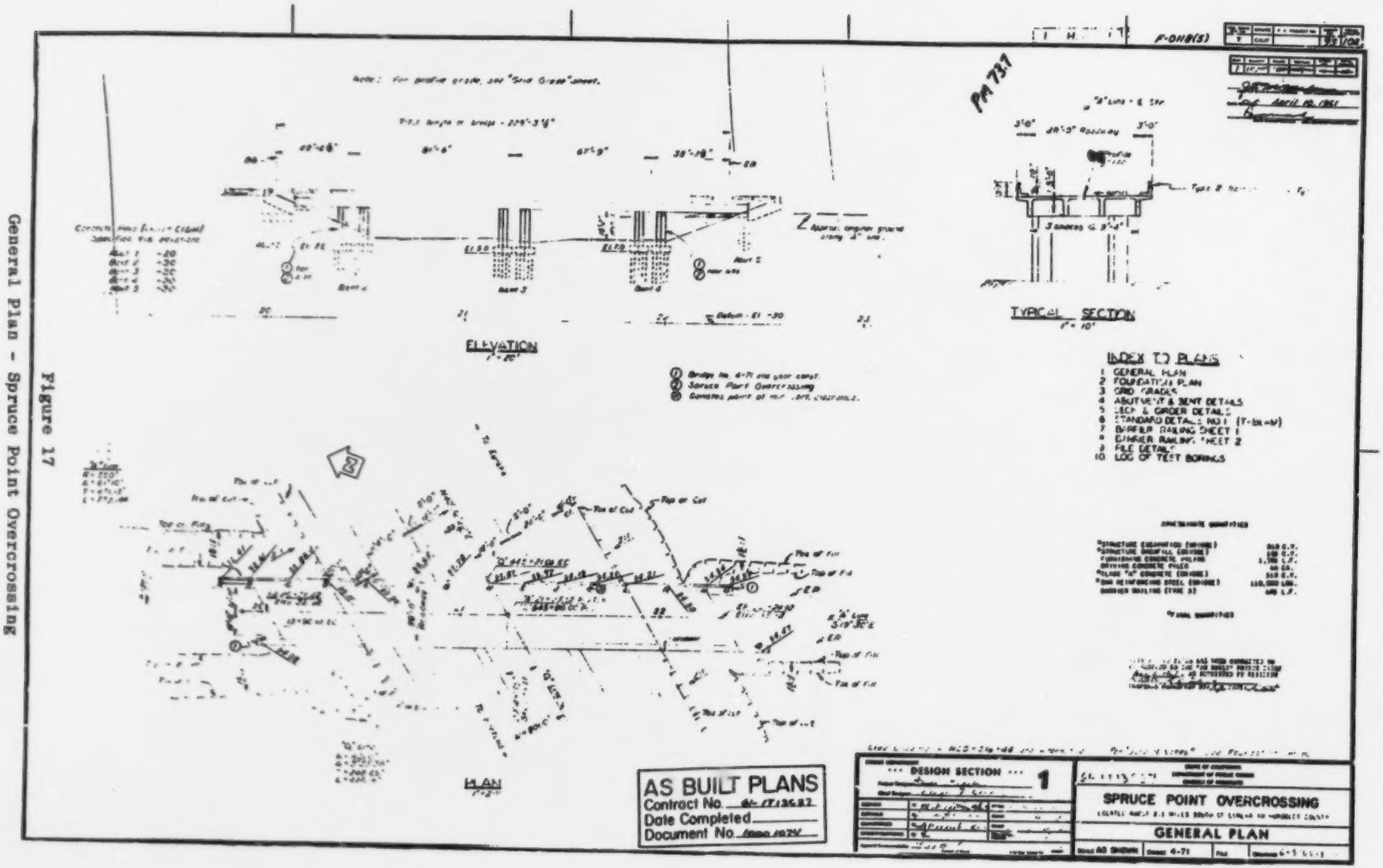

Figure 17 General Plan - Spruce Point Overcrossing

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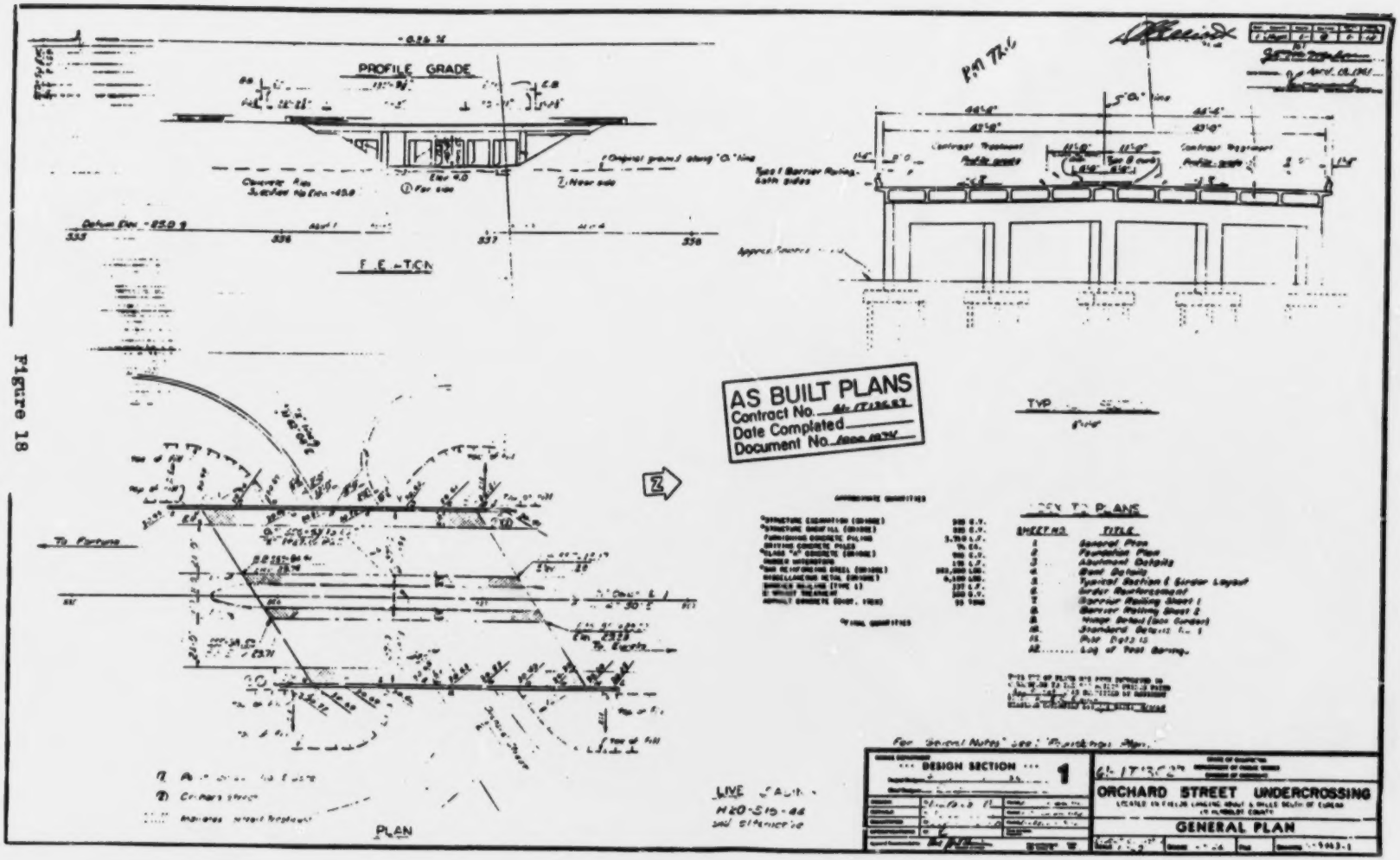

Figure 18 General Plan - Orchard Street Undercrossing

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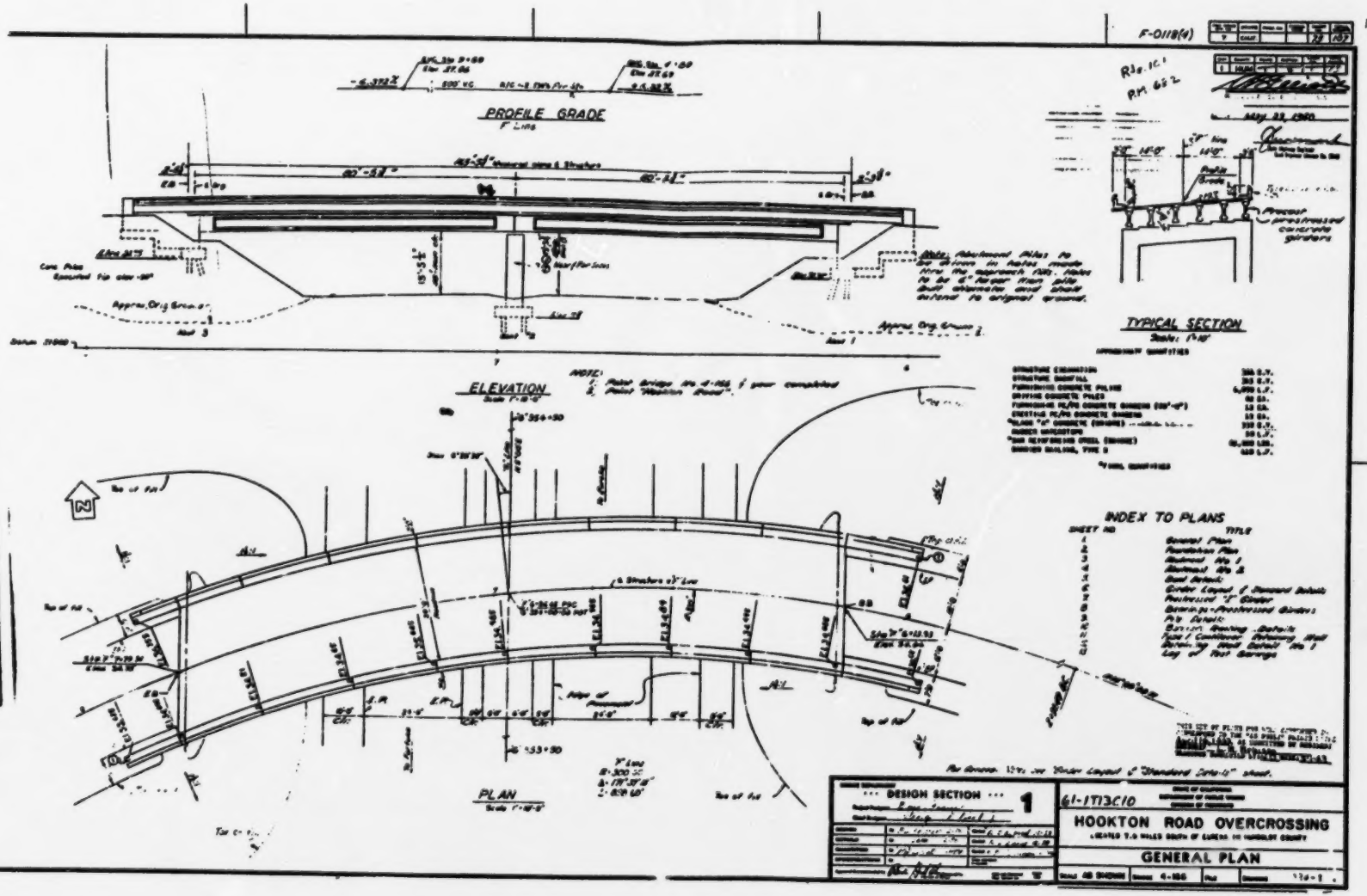

Figure 19 General Plan - Hookton Road Overcrossing

Document Number

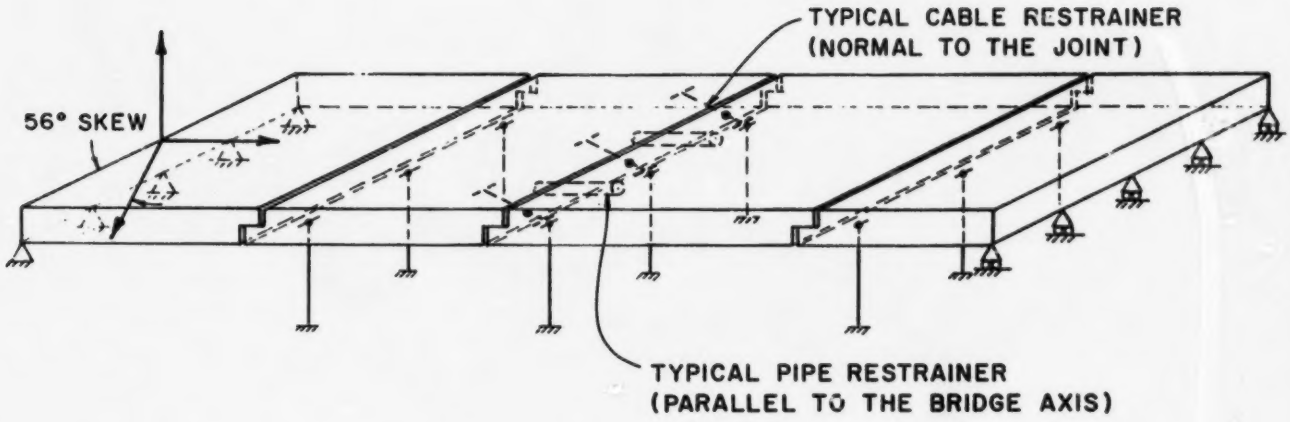

Figure 20

Retrofitting at Bents

APPENDIX - RECORDED GROUND MOTION RECORDS

EUREKA, CA or TRINIDAD, CA - OFFSHORE 08nov80

(USGS) origin time 02:27am pst
epicenter 40.93n, 124.48w
or farther west
magnitude 6.6 to 7.1

(NEIS) epicenter 41.1n, 124.3w
magnitude 7.0
depth 19 km
7.0 Ms

(UCB) epicenter 41.0n, 124.6w
magnitude 7.0

aftershocks 4.5 or greater
08nov80
1) 02:48 am pst 4.6
2) 02:53 am pst 4.8
3) 03.20 am pst 4.6
4) 02:48 pm pst 4.8
5) 08:10 pm pst 5.2

station number	name	coord	site character	distance	acceleration	
1022	Eureka federal bldg CDMG Sta 327	40.80n 124.16w	prtlly cnsl sediments	30 km	78 up 348	0.12 g 0.05 0.10
1498	Rio Dell Pntr St Ovrpss CDMG Sta 324	40.50n 124.11w		56	270 up 360	0.15 g 0.03 0.06
1249	Cape Mendocino CDMG Sta 156	40.35n 124.35w	rock	65	< 0.05 g	
1398	Petrolia general store CDMG Sta 156	40.32n 124.29w	alluvium	69	98 up 08	0.07 0.02 0.06
1395	Willow Creek CDMG Sta 146	40.94n 123.63w	alluvium	71	< 0.05 g	
1261	Cresent City CDMG Sta 17	41.77n 124.19w	alluvium	97	< 0.05 g	
1277	Shelter Cove 1 CDMG Sta 41	40.04n 124.06w	rock	104	46 up 316	0.06 g 0.04 0.04
1278	Shelter Cove 2 CDMG Sta 40	40.03n 124.06w	alluvium over rock	105	226 up 136	0.11 g 0.04 0.16

In addition, the Humbolt Bay Nuclear Power Plant has strong motion instruments operated by PG&E.

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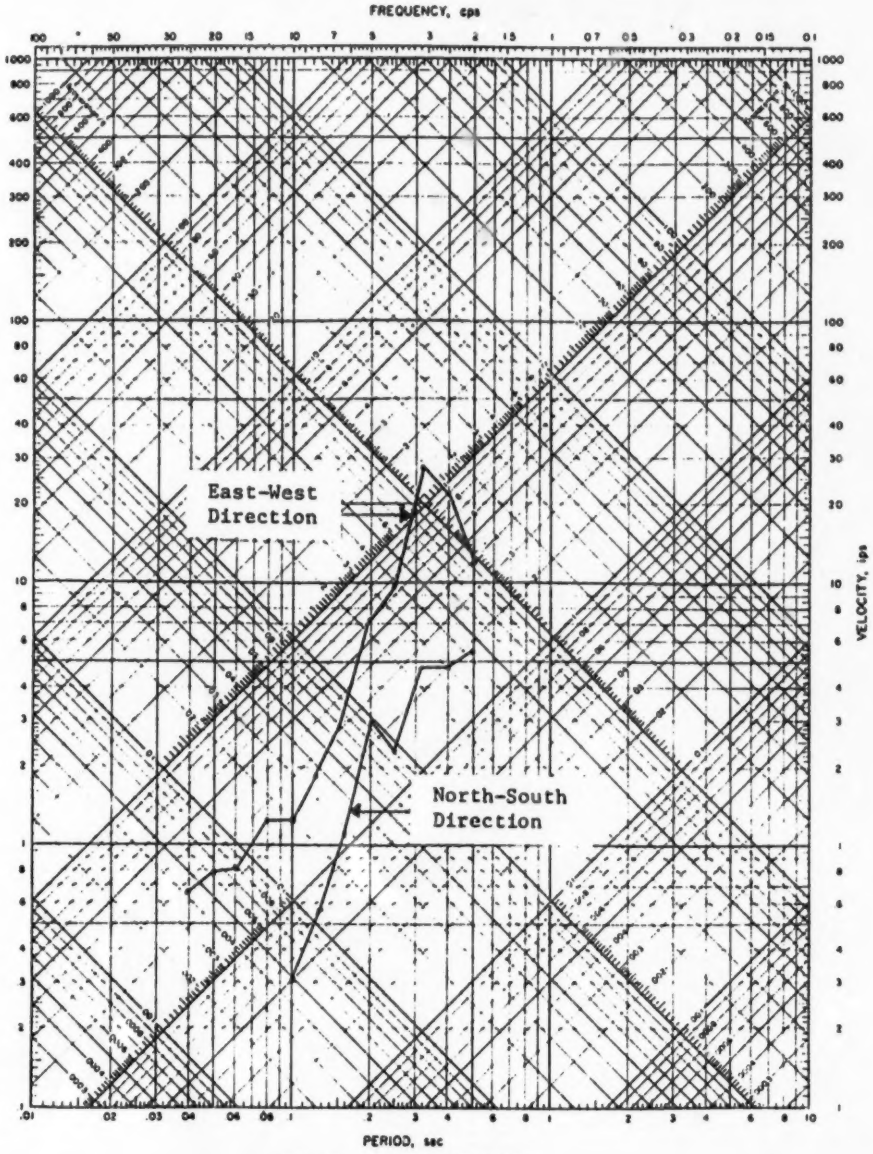

PEAK SHOCK SPECTRA
Pacific Gas & Electric Humboldt Bay Power Plant
Trinidad-Offshore Earthquake of November 8, 1980
(Courtesy Mr. Otto Steinhardt, PG&E, San Francisco, CA)

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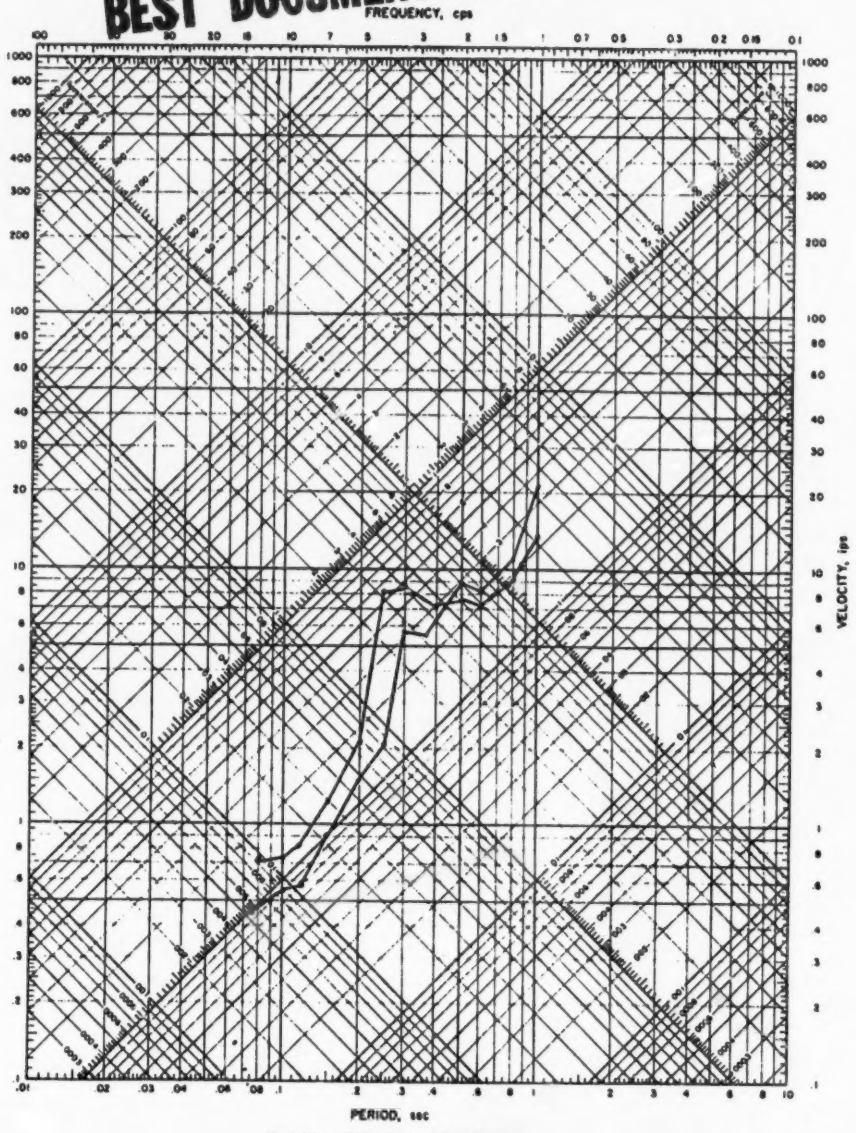

PEAK SHOCK SPECTRA
Loleta Overcrossing
Trinidad-Offshore Earthquake of November 8, 1980
(Courtesy Mr. James Gates, Caltrans, Structures Design)

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.